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Challenges and opportunities in tropical concreting

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Abstract

Much of the technical knowledge on concrete as a construction material originates from research and experiences in temperate regions of the world. Unlike other construction materials, such as steel, timber and masonry, the behaviour of concrete is much more influenced by its service environment, temperature and relative humidity. The transfer of knowledge on concrete, developed for temperate climate, calls for satisfactory adjustment when adopted for tropical climates. These are challenges which also create opportunities for innovative solutions. Firstly a brief review of the main effects of tropical temperatures on properties of fresh and hardened concrete are presented. Methods to mitigate the less desirable influences of temperature are considered. Some advantages of the tropical ambient temperature on specific applications are described. The urgent need to gather information from monitoring durability performance in tropical exposure environments in order to calibrate future performance based design approaches for durability is emphasised, as these are expected in the coming years. A significant period of local experience and monitored performance data in tropical environment is needed to provide input to calibrate these design models for tropical exposure when they have reached international consensus.

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*Keywords:*Tropical concreting, temperature effects

1. Introduction

One of the early publications on the effects of warmer climate compared to temperate climate was entitled "Concrete in Hot Countries" by STUVO [1], Netherlands member group of FIP at the FIP Congress in New Delhi.

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The examples on the influence of temperature on the initiation time for corrosion due to chloride ingress shows that at $30^{O}C$ it is approximately one-third of that at $10^{O}C$ based on Fick's second law. Since then a much better appreciation of the effects of temperature on fresh and hardened performance has been gained from both experience and research studies. However, with the rapid changes in materials technology as well as the move towards high performance concrete in terms of strength, consistence and durability as well as the drive for more sustainable concrete construction, a review of the progress in these areas is useful. The aim is to provide more economic solutions to meet the added difficulties in tropical climatic conditions. Such challenges also create opportunities for innovative approaches to achieve the different high performance requirements with more cost-effective solutions.

2. Effect of Temperature on Fresh Concrete

Major considerations for effects of temperature on properties of fresh concrete are:

(a) Initial consistence and rate of loss of consistence with time (e.g. transportation)
(b) Stiffening time (potential cold joint formation)
(c) Potential plastic settlement
(d) Potential plastic shrinkage
(e) Initial concrete temperature (at time of placing)

Each of the above is discussed in the following sections.

2.1. Consistence and Stiffening Times

Both the initial level of consistence at the end of mixing as well as the subsequent rate of loss in consistence over time are significantly influenced by higher ambient temperatures. Not only are constituent materials at higher storage temperatures, the rate of initial hydration during mixing is also higher. The advance of chemical admixtures from the early 1930 to currently available engineered admixtures has provided economic solutions to most of these issues. Very high consistence up to flowing concrete, like SCC, is available to meet special site situations in construction and is used extensively in some sectors of the concrete industry, e.g. precast post-tension elements. Retention of high consistence and desired delay in setting are both enabled by the addition of appropriately formulated admixtures. Hence large volume pours of over 10,000 m^3 can be placed in a single continuous operation. Typically this is evaluated through the carrying test based on reaching a prescribed level of penetration resistance of wet-sieved mortar from the concrete, e.g. ASTM C 403-08 [2]. Even though the test is not directly related to potential cold joint formation, it is indicative of the time since the mixing of the concrete to the time when vibration is no longer able to homogenize newly placed fresh concrete with previously-placed concrete. This leads to the formation of a cold joint affecting the integrity of the structural element, particularly in its performance in shear. A more desirable approach that can indicate directly the potential for formation of cold joint has yet to be developed.

2.2. Potential Plastic Cracking

Due to higher tropical ambient temperatures the addition of set-retarding admixture is necessary for large volume casting, e.g. thick raft foundations. During the plastic stage of fresh concrete until its initial setting time, concrete tends to settle downwards. If this downward movement is hindered, e.g. by top reinforcement bars in a raft foundation or a deep transfer girder, a crack may develop directly over the location of bars, indicated by a crack pattern similar to that of the bars. Although it is possible to close the cracks if noticed before the concrete sets, it is better to mitigate the potential by avoiding longer-than-necessary retardation times and by having a more cohesive concrete to minimise settlement. Such cracks can occur within the first hour or the next couple of hours after placing.

When fresh concrete is exposed after finishing and bleeding tends to end, continued loss of moisture from the surface due to evaporation may lead to potential plastic shrinkage cracking. The rate of evaporation may be estimated from the equation by Uno [3]:

$$Evaporation\ Rate\ \left(kg/m^2h\right) = 5\left\{\left(T_C + 18\right)^{2.5} - R\left(T_a + 18\right)^{2.5}\right\}\left\{V + 4\right\}x10^{-6} \quad (1)$$

where

T_C = concrete (water surface) temperature (°C)

T_a = air temperature (°C)

R = relative humidity of air (%)

V = wind velocity (km/h)

The critical rate for plastic shrinkage has been proposed as 1kg/m^{2}h (0.20lb/ft^{2}h) for Portland cement concrete. It is reduced to 0.7 kg/m^{2}h for Portland cement concrete with more than 15% pozzolan and 0.5 kg/m^{2}h for Portland cement concrete with more than 5% silica fume, as stated in Concrete Construction [4]. In late afternoons in tropical climates, the air temperature lies around 32^O to 35^{O}C, relative humidity drops to about 60% (0.60), together with a wind velocity of 10 km/h and the concrete temperature reaching 5^{O}C higher than the air due to mixing and transportation with agitation, the critical rate of 1 kg/m^{2}h is easily reached. Hence, early curing and mitigating methods to reduce wind velocity (wind breakers) is required, e.g. casting floor slabs at higher levels of tall buildings.

2.3. Initial Concrete Temperature

The concrete temperature at the end of mixing is a few degrees higher than the ambient temperature as most constituents are at least at ambient temperature and cement in a silo lies at a much higher temperature. By the time the concrete is delivered to the site, additional gains in temperature are caused by hydration of cement and agitation energy during the transportation from plant to site. Typically concrete at time of discharge is around 5^{O}C or higher above the ambient temperature. Based on the heat capacity of constituent materials (mass x specific heat) and their temperatures, initial concrete temperature is given by the equation:

$$T_o = \left\lfloor M_c H_c T_c + M_w H_w T_w + M_{fa} H_{fa} T_{fa} + M_{ca} H_{ca} T_{ca} - M_i F_i \right\rfloor / \left\lfloor M_c H_c + M_w H_w + M_{fa} H_{fa} + M_{ca} H_{ca} + M_i H_w \right\rfloor \quad (2)$$

where

M = mass (kg)

H = specific heat of ingredient (kJ/kg°C)

T = temperature (°C)

F_i = latent heat of fusion of ice (335 kJ/kg)

Subscripts: c, w, fa, ca and i represent cement, water, fine aggregate (SSD), coarse aggregate (SSD) and ice, if used.

Typical values of specific heat (kJ/kg) for cement = 0.88, fine and coarse aggregates = 0.75 and water = 4.18. In tropical climates, the temperature of cement stored in silo is usually between 40° to 60°C and the other constituent materials at around ambient temperatures of 25°C to 30°C when stored under the shade and sheltered from the sun. Without the use of chilled water or ice as partial replacement of batch water, initial temperature at the time of discharge is typically between 32°C to 35°C. This level of initial temperature poses difficulties to meet limiting peak temperatures of 70^{O}C and above where there is risk of delayed ettringite formation, a potential cause of thermal induced cracking (considered in the next section).

3. Effect of Temperature on Hardened Concrete

The effect of temperature on the rate of cement hydration and hence the rate of strength development over the duration of moist curing is well-known. The higher ambient temperature in tropical regions promotes faster strength gain with time. However, it is important to note that higher temperatures at the time of setting have the opposite effect on later age strength development, i.e. faster hydration during setting leads to a lower strength gain at later ages [5]. Hence, a slower rate of hydration during settling leads to a longer period of significant strength development. Concrete set and cured at a lower temperature will eventually overtake one set and cured at a higher temperature in strength. The cross-over point is usually after 90 days between temperatures of temperate climate

(20^{o}C) and tropical climate (30^{o}C). This implies that for equal 28-day strength adopted in design, the long term quality of tropical concrete is expected to be lower than concrete in temperature climate. Thus together with a faster rate of deterioration in severe exposure conditions due to higher tropical temperatures, the durability of concrete with the same nominal 28-day strength will show distress at an earlier time if designed for the same nominal exposure conditions based on recommendations adopted for temperate climate, e.g. European practice, BS EN 206-1[6]. Tropical concreting practice should factor in the experience of earlier than expected distress in the Middle East even though the conditions in tropical climate are less severe. Examples of the effect of higher ambient temperature on durability performance of concrete were shown in a presentation by Otsuki et al. [7]. The carbonation coefficient is higher by a factor of 1.2 to 1.3 for temperatures of 30^{o}C compared to 20^{o}C. The estimated incubation period, propagation period, acceleration period and life due to chloride-induced corrosion in reinforced concrete is about 70% in tropical climates, e.g. South East Asian countries, compared to temperate regions such as Japan.

3.1.Effect of High Temperature on Potential Delayed Ettringite Formation

The current interest in high strength concrete for structural elements of large dimensions is on the increase e.g. columns in tall buildings and transfer girders. The need for thick raft foundations or deep pile caps for such development is also facing the issue of exceeding the peak temperature limit of 70^{o}C for which potential delayed ettringite formation (DEF) may occur. Unlike temperate countries where even in summer, the average ambient temperature is only around 20^{o}C, in tropical climates the average ambient temperature throughout the year is about 30^{o}C. Even when low heat of hydration cement is used at typical characteristic heat of hydration at 7 days not more than 270 kJ/kg, e.g. CEM III/B-LH up to 80% ggbs or CEM II/B-V up to 35% fly ash, the adiabatic temperature rise is still of the order of 10^{o}C/100 kg/m^{3} cement content. Under this scenario, cement content is limited to not more than 400 kg/m^{3} in order for the peak temperature to not exceed 70^{o}C. For water contents as low as 140 kg/m^{3}, the water/cement ratio for concrete is 0.35. Thus this limits the level of compressive strength of concrete that can be adopted for design. Although partial replacement with of 5 to 10% silica fume in the total cement content, compressive strength of C60/75 and above can be achieved when a margin of 10-15 MPa is added to arrive at the target mean strength in production. The available solutions have to go beyond the use of chilled water and/or crushed ice to lower the initial concrete temperature at the time of placing to below 20^{o}C. Pre-cooling of aggregates or injection of liquid nitrogen into fresh concrete and post-cooling by circulating cooled water through embedded pipes are not only costly solutions, but non-ubiquitous due to the infrequent demand for such technology. The initial investment for such systems is high and unless the need is in general demand, the investment return is a major factor which deters their local availability.

3.2. Potential Early Thermal Cracking

In addition to the issue of potential DEF, there is also the need to minimize potential occurrence of early thermal cracking. This arises from the low heat loss from the interior of a thick section and the much faster cooling of the external surface zone. This temperature differential, together with the degree of boundary restraint, may lead to tensile strains at the surface exceeding the tensile strain capacity of the concrete (depending on its coefficient of thermal expansion for different types of coarse aggregate). Often the specification calls for the maximum temperature differential not to exceed 20^{o}C. In Table 2 of BS 8110: Part 2 [8] the value of 20^{o}C is associated with the use of gravel aggregate and a restraint factor of 0.36. This is based on the observation that in the UK "experience has shown that by limiting temperature differentials to 20^{o}C in gravel aggregate concrete, cracking can be avoided". For other types of aggregate and different restraint factors, e.g. Table 3.3 in BS 8110: Part 2 [8], guidance in Table 3.2 provides other estimated "limiting temperature changes to avoid carking".

3.3.Mitigating Peak Temperature and Potential Early Thermal Cracking

In order to mitigate both peak temperature and potential early thermal cracking, Tam et al. [9] proposed a cost-effective strategy based on the "sandwich" concept in the casting of a thick raft foundation. In this approach, the total thickness of the raft is divided into three zones: an interior zone of about 30% to 40% of its thickness separating two other outer zones each of about 20% to 30% of the total thickness. Only the interior zone is placed with the lowest achievable fresh concrete temperature (including use of chilled water and/ or ice) but with both outer zones with normal fresh concrete temperatures. The initially cooler concrete will at later ages overtake the higher outer zone concrete temperatures as hydration takes place over time. However, it has to cancel out the initial temperature differential (opposite to the final differential) before building up the eventual temperature differential. In this way, the final temperature differential is reduced by the initial opposite difference in temperatures. Hence, not only is there a cost-reduction due to diminishing the amount of chilled water and/or ice for the total concrete volume, it also provides an improved technical solution in developing a lower temperature differential. The proposed method of dividing the total thickness of a thick raft foundation into multiple layers of 300 mm to 400 mm each enables each layer to be placed within a 2 to 4 hours by multiple discharge points for simultaneous casting over a wide area (in plan) of the large volume pour. Thus the total delay in setting is minimal. If site constraints do not permit multiple discharge points, then the required extended delay in setting can be determined and suitable retardation time provided in developing the concrete composition with appropriate dosage of set-retarding admixture. However, the top most layer does not need the longer retardation to minimize potential plastic settlement and plastic shrinkage cracking. This can be achieved by adjusting the dosage of two admixtures: one with a high retardation and normal plasticizing effect, the other a super-plasticiser with high a water-reducing capacity but low retardation. The casting in horizontal layers of 300 mm to 400 mm thickness ensures proper compaction. This is unlike the common practice of placing concrete from a single discharge point and building it up to full thickness which results in the sloping front spreading wider and wider away from it. This large exposed sloping surface calls for a very long retardation time to prevent potential cold joint, often 8 to 10 hours. Such concrete placed up to full thickness will still be hours before setting, leading to potential plastic settlement cracking whilst the rest of the volume of concrete is being placed to cover the whole area in plan. In addition, the long exposure time may lead to potential plastic shrinkage cracking if the bleed water at the top surface has fully evaporated.

In order to demonstrate the degree to which the planned approach is expected to perform, a carefully planned mock-up using the actual designed concrete for each of the multiple layers (differing in initial temperature and setting time) is recommended. Both the peak temperature and the maximum temperature differential are monitored in the mock-up. A cross-section of 2.5m^2 and the actual thickness are adopted with the planned number of layers (odd number preferred as the peak temperature is likely to be in the middle layer). The bottom is provided with the same thickness and type of concrete as the lean base and the top surface without any insulation, unless otherwise required by the results of the mock-up. With this cross-section, each layer of 300mm to 400mm requires about 2 m^3 so as to be representative of actual delivery for the actual pour. At least 200mm thick polystyrene foam insulation is provided on the four sides of the mock-up formwork to simulate the effect of the much larger plan area of the actual pour. This approach has been applied successfully in many projects over the past fifteen years involving many thousands of cubic metres of substructure concrete in tall buildings and MRT tunnels in Singapore (Fig. 1 to 4).

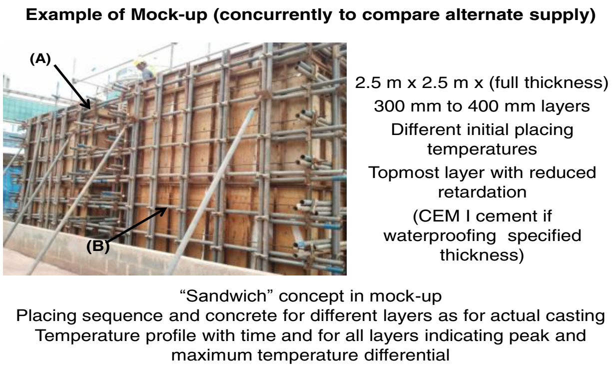

Fig. 1. Mock-up formwork

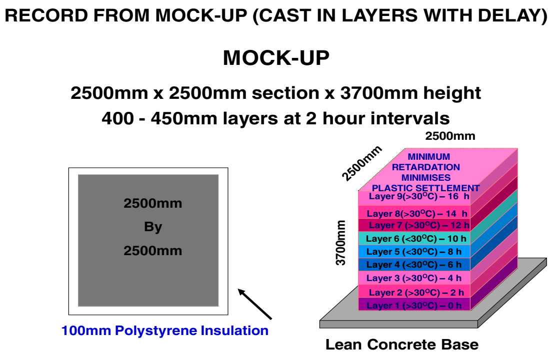

Fig. 2. Casting sequence of layers for mock-up

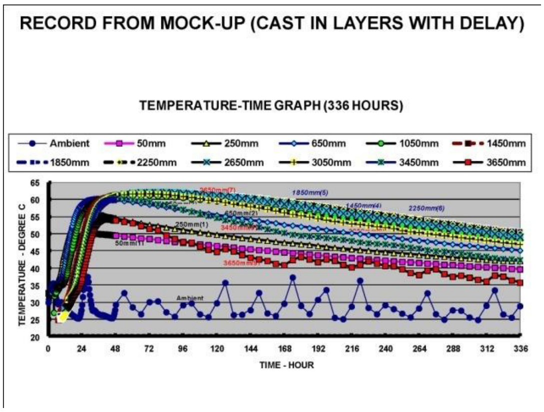

Fig. 3. Example of recorded temperature history

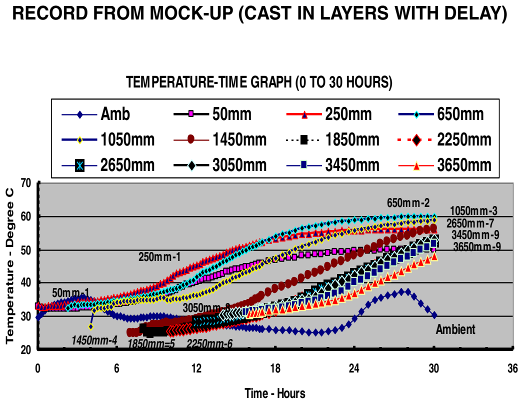

Fig. 4. Monitoring of early stage of placing

4. Challenges and Opportunities

Every challenge in design and construction offers the opportunity for innovative solutions. The case of casting a thick raft foundation has been presented as an example. Another opportunity of innovative solution to a challenge is illustrated in the forecourt structure of the Kuala Lumpur International Airport (KLIA). This is based on recognizing the near constant ambient temperature throughout the year in the tropical region for which seasonal thermal movement is insignificant in tropical climate.

4.1. KLIA forecourt structure of the Kuala Lumpur International Airport

The construction of the forecourt structure of the KLIA of 440m long and 34m wide with a steel roof was constructed without any provision for movement joints (Fig. 5).

Fig. 5. KLIA Forecourt building

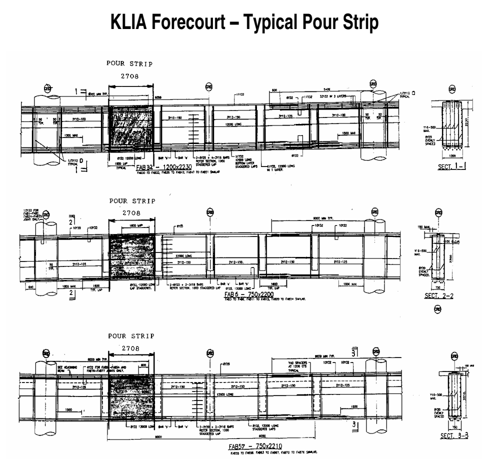

Fig. 6.Redesigned locations for pour strips

Although the original design incorporated two transverse joints at 150 m centres, the challenge is to redesign and construct without such transverse expansion joints due to serviceability issues associated with heavy traffic loads, construction difficulties with complicated details, and the need for regular maintenance. This is made possible when it is realized that with near constant mean monthly temperatures, there is no need to provide for seasonal thermal movement commonly practised in temperature countries. Hence, this leaves only the long term shrinkage as the only component of movement as this is a RC structure without any pre-stressed element. The finally adopted construction approach was to allow most of the estimated long term shrinkage to take place before the structure was made into a single long building by means of four [4] numbers of infill pours strips which were only cast when 81% of the estimated long term drying shrinkage of the concrete was realised as shown through monitoring of actual shrinkage on site during the construction stage. These four pour strips were at 80 – 100 m centres (Fig. 6). Ultimate drying shrinkage was first estimated from known prediction formulae and compared with site monitoring results. Details of the monitoring system and locations of measurements are available from Yabe et al. [10]. A summary of the comparison is shown in Table 1. The forecourt structure has been in service satisfactorily for nearly 20 years as a continuous structure of 440 m in length without any movement joint. Thus a reduction in both construction and maintenance cost is achieved with the application of better understanding of concrete performance involved and by adopting an appropriate alternate design as well as an innovative construction solution.

Table 1 Estimated and in-situ drying shrinkage of concrete

Model/ Member	Drying shrinkage (microstrain)						
	Edge Main Beam		Secondary Beam		Main Beam		Overall
	Rectangular Beam Model	T-Beam Model	Rectangular Beam Model	T-Beam Model	Rectangular Beam Model	T-Beam Model	T-Beam Model
BS 8110, Part 2, 1985	150	166	167	189	NA	146	172
ACI 209.R-82	158	218	283	346	90	144	273
CEB – FIP	165	171	163	178	154	162	172
T.C. Hansen et al	243	333	433	529	140	222	420
In-situ shrinkage	243 – 262		193 – 313		260 – 340		193 – 340

4.2. Punggol Sea Wall

The long term durability of infrastructure is another major challenge in the higher ambient environment of tropical climate. The effect of the fast rate of chemical reaction that impact on the durability intended working life of concrete structures due to higher tropical temperatures throughout the year provides a challenge to prolonging service life for as long as possible, extended further by repairs or replacement of protective system. An example of providing for a highly aggressive marine exposure condition is the adoption of a sacrificial left-in-place formwork of high durability in the Punggol Sea Wall (Fig. 7). It is a retaining seawall structure of 7 km. A 30 mm thick ferrocement precast unit of 2.03 – 2.60 m by 3 m width was cast into the region where the 5.6 m high seawall is exposed to the tidal range (Fig. 8). Details of the materials used in both the ferrocement panels and the seawall are described in Tam et al. [11]. This is a special case of almost close to"Avoidance-of-deterioration method" approach as the concrete behind the ferrocement left-in-place formwork is protected from both ingress of chemicals and hence potential corrosion of its embedded reinforcement steel bars. The seawall structural has been in use for more than a decade and is likely to have at least a 50 years' intended working life and with potential for *insitu* restoration of the ferrocement to extend its working life.

EXAMPLE OF DURABILITY PROVISION

Fig. 7. Punggol seawall – length 7 kilometers

Fig. 8. Precast sea wall with ferrocement panel

5. Concluding Remarks

The knowledge on concrete and its constituent materials has come a long way from the early 1930s when the use of concrete in buildings was first codified in the UK and USA. The introduction of chemical admixtures firstly as air-entraining agent and followed by water-reducing and set-retarding through to the latest engineered super-plasticizers has enabled enhancement in concrete performance. These include the development of high performance concrete in terms of high strength concrete, high consistence concrete and high durability concrete. Understanding of basic mechanisms involved in deterioration processes have also increased over the years. One can look forward to the eventual codified performance-based approach in quantitative terms in the near future. In order to adopt the basic principles provided in quantitative models for durability design, calibration based on monitored performance in local climates for materials used calls for provisions in national annexes and supporting guidance standards. Hence, there exists the urgent need to establish centres to carry out these tasks, as at 10 to 20 years of performance data well correlated with *insitu* concrete quality and documented exposure history will be required for such purposes.

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